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MODELING TECHNIQUES FOR RC-FRAME SYSTEMS WITH INFILLS

M. Leipold* **and J. Schwarz**

* *Graduiertenkolleg 1462*
Bauhaus–Universität Weimar
Berkaer Str. 9
99425 Weimar
E-mail: mathias.leipold@uni-weimar.de

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Abstract. *There are many different approaches to simulate the mechanical behavior of RC frames with masonry infills. In this paper, selected modeling techniques for masonry infills and reinforced concrete frame members will be discussed – stressing the attention on the damaging effects of the individual members and the entire system under quasi–static horizontal loading. The effect of the infill walls on the surrounding frame members is studied using equivalent strut elements. The implemented model consider in–plane failure modes for the infills, such as bed joint sliding and corner crushing. These frame member models differ with respect to their stress state. Finally, examples are provided and compared with experimental data from a real size test executed on a three story RC frame with and without infills. The quality of the model is evaluated on the basis of load–displacement relationships as well as damage progression due to quasi–static horizontal loading.*

1 INTRODUCTION

Increasing computational power and sophisticated software entice engineers to rise the complexity of the structural modeling in civil engineering for gaining more accurate results. But mainly two aspects seem to be notable. Once, more complex models mostly demand a higher number of input parameters in order to provide more accurate results. Additionally, the determination of input parameters is accompanied with uncertainties, which consequently affect the output parameters. This leads to the question of how reasonable such strategy is, what can be expected from different models and how to evaluate these models. In this paper preliminary answers are given for a reinforced concrete structure with and without masonry infills. Results are presented for numerical simulations considering different models for the structural elements. This chosen structure is of particular interest, because it had been built and tested in real size by DOLŠEK AND FAJFAR [1]. Their modeling strategy including simulation results and several construction details are used for comparison. A qualitative evaluation is done for the different formulations of the structural models. Relying on that, further extensions to evaluate structural models in a more general way will be discussed.

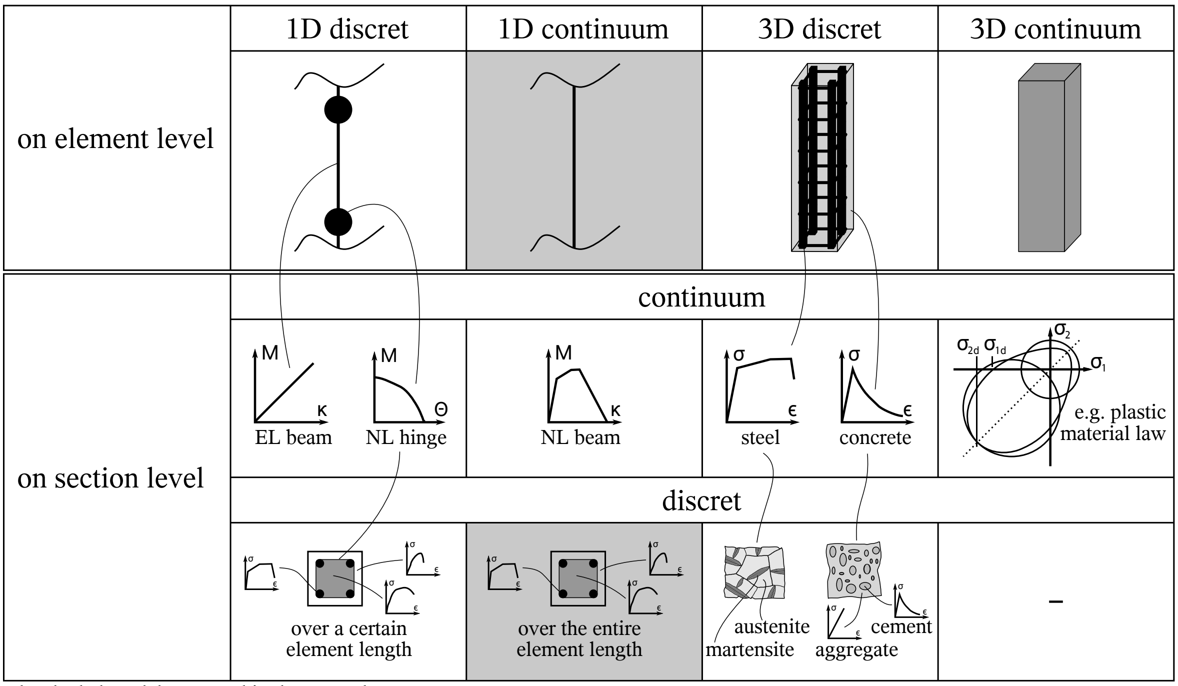
2 STRUCTURAL ELEMENTS OF RC FRAME SYSTEMS

Capturing the mechanical behavior of RC frame systems subjected to lateral loading and considering damaging effects are still challenging tasks in civil engineering. Reasons for the problems originate from the combination of two construction techniques – one is the reinforced concrete frame members, and the other is the infill mostly masonry – into one interactive system. In the elastic range, such a system is relatively simple to model. Considering different damaging mechanism of reinforced concrete and masonry members additional effort is required to model the system and to get reliable results. RC frames are mainly designed to carry loads by bending, and hence they are relative flexible systems. In contradiction to this, infills act as relative stiff diaphragms because of their shape and the type of loading by the surrounding frame. Of course, infilled or partial infilled RC frames are stiffer than bare ones, furthermore the damaging behavior can change completely. The degree of mechanical interaction and reciprocal damaging action of infill and frame depends on the following conditions:

- Properties of the frame members as stiffness and plastic behavior – location of hinging – which lead to different loading states and restoring forces in the infill.
- Properties of the masonry block or brick units as Young's modulus, strength, perforation ratio, perforation shape, size, porosity, moisture content.
- Properties of the masonry mortar as strength, adhesion strength, moisture content, tendency to shrink.
- Bond properties between units and mortar as, adhesion strength, manner of assembling, workmanship.
- Construction properties of the panel as openings, brickwork bond, workmanship – surface contact characteristic between infill and frame.

This relative high complexity leads obviously to a conflict in the aim to simulate such structures numerically concerning the practical aspects. On one side, it has a lot to commend to form a

Table 1: Models for structural elements used in RC frame structures to capture damaging effects



The shaded models are used in the example.

complete model for the entire structure with elements that are able to catch nearly all known effects. In such cases, a huge computational effort is inevitable. Moreover, it is most likely to get no result at all, because of numerically instabilities, the needed time or the difficulty to interpret the obtained results. On the opposite side are simulation strategies, which try to avoid the problems aforementioned by preferring relatively simple models for the structural elements. These models have only selected capabilities to describe the phenomenological behavior of a structure. A substantial disadvantage is the need to know the occurring damaging mechanism a priori in order to implement these extra features. More inside of the behavior the most appropriate modeling technique could be obtained of instrumental testing and reinterpretation of the observed response under horizontal action (cf. LANG ET AL. [2], ABRAHAMCZYK ET AL. [3]). In the following selected modeling techniques for reinforced concrete frame members and masonry infills are introduced and discussed.

2.1 Elastic Beam Elements with Allocated Non-Linear Bending Hinges

Beams and columns are modeled using concentrated plastic hinge models or distributed plastic hinge models to represent important failure characteristics of reinforced concrete beam and column components subjected to lateral loading. The element model is not capable of representing inelastic response continuously along the member length. The yielding areas are restricted to the component ends where the hinge models are placed as illustrated in Table 1. This models type are commonly recommended for use in codes as FEMA 273 [4] to get an overall impression of the non–linear failure mechanism of a building. The hinges themselves are usually defined by a moment–rotation relationship or moment–curvature relationship over a certain length. Where nonlinear response is expected in a mode rather than flexure, these models can be established to represent these effects.

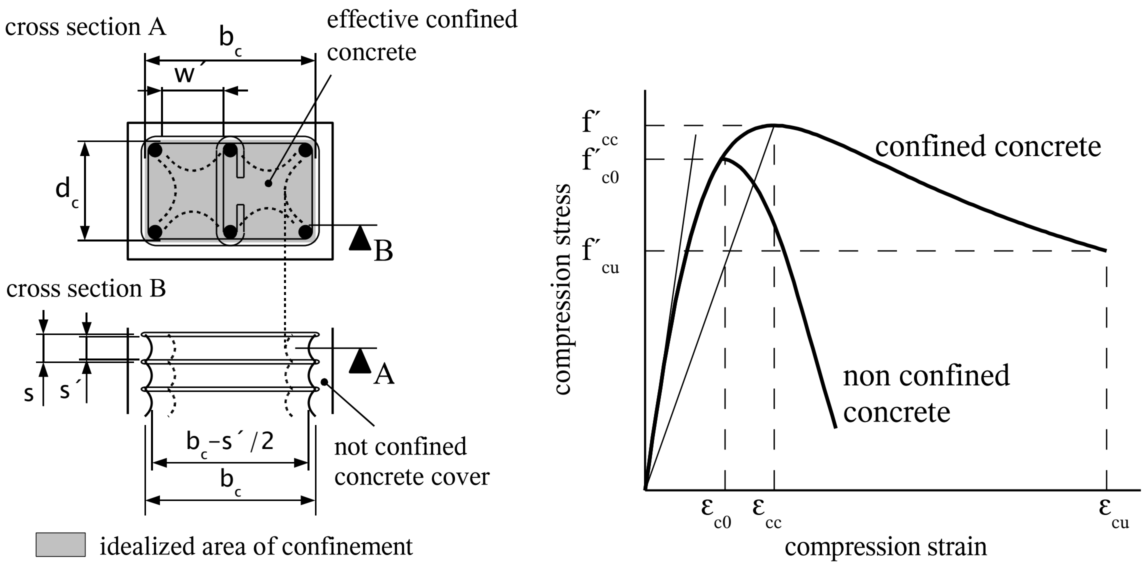


Figure 1: Effect of lateral confinement in reinforced concrete members subjected by normal load according to MANDER ET AL. [5]

2.2 Continuously Non-Linear Properties with Beam or Column Elements

Instead of restricting the occurrence of damage to chosen areas by moment–curvature relationships, these zones could be extended over the whole element length. So it is possible to get information about the damage distribution for the element. This behavior depends not only on the mechanical properties of the main reinforcement and the concrete, but also on the normal force condition and the lateral stiffness of the shear reinforcement influencing the damaging response. Besides appropriate moment–curvature relationships, there is the possibility to model the cross section in a discrete manner by distinguishing between main reinforcement, core concrete and concrete cover to consider their influence on the structural element separately. An accordingly method is proposed in MANDER ET AL. [5], which enables to calculate an equivalent uniaxial material law for the core concrete depending on the limitation of the Poisson–Effect due to stiffened lateral reinforcement as shown in Fig. 1. That seems to be not only a simple and reasonable way to take spatial stress states into the calculation, but also it enables the identification of damaged areas observing strain conditions of the cross section. The consideration of the confining effect in an uniaxial material law for concrete is presented in MANDER ET AL. [5]. The relations are given as follows,

$$f_c(\epsilon_c) = \frac{f'_{cc} * x * r}{r - 1 + x^r} \tag{1}$$

$$f'_{cc} = f'_{c0}\left(2.254\sqrt{1 + \frac{7.94 * f'_l}{f'_{c0}}} - 2\frac{f'_l}{f'_{c0}} - 1.254\right) \tag{2}$$

Equation 2 calculates the maximum concrete strength f'_{cc} due to the confinement. The value of f'_{c0} is the maximum strength of the non–confined concrete and correspond to results of standard compression test ($H = 20\,cm$, $D = 10\,cm$ specimen). Other input parameters are given by Eqs. 3 to 5.

$$f'_l = 0.5 * k_e * f_{yh} * \rho_s \tag{3}$$

$$k_e = A_e/A_{cc} \tag{4}$$

$$A_e = (b_c b_d - A_i) * \left(1 - \frac{s'}{2b_c}\right) * \left(1 - \frac{s'}{2d_c}\right) \tag{5}$$

Equation 5 shows the area of effective confined concrete and is only valid for rectangular shaped lateral reinforcement. The parameter b_c, b_d and s' depends on geometrical properties of the section, see Fig. 1.

$$A_i = \sum_{i=1}^{n} \frac{{w'_i}^2}{6} \tag{6}$$

Equation 6 describes the non–confined area enclosed by the hoops founded by the arch action developing in the concrete. Based on the assumption this arch is locally supported by the reinforcement and the simplification of it shape is parabolic.

$$A_{cc} = A_c - A_{SL} \tag{7}$$

$$A_c = b_c * d_c \tag{8}$$

A_{cc} describes the area circumscribed by the centerline of the lateral reinforcement (A_c) without the area of main reinforcement A_{SL}, see Eq. 7 and Fig. 1.

$$\rho_s = \rho_x + \rho_x \tag{9}$$

$$\rho_x = \frac{A_{sx}}{s * d_c} \tag{10}$$

$$\rho_y = \frac{A_{sy}}{s * b_c} \tag{11}$$

The lateral reinforcement ratio is considered by ρ_s. In Eq. 10 A_{sx} is the entire amount of reinforcement distributed over the distance s runs in x–direction. With this the y–direction is defined as d_c, see Fig. 1. For A_{sy} (Eq. 11) it is inverse. The yield strength of the lateral reinforcement is given by f_{yh}. In MANDER ET AL. [5] $\epsilon_{c0} = 0.002$ is labeled together.

$$x = \epsilon_c/\epsilon_{cc} \tag{12}$$

$$\epsilon_{cc} = \epsilon_{c0}\left[1 + 5\left(\frac{f'_{cc}}{f'_{c0}} - 1\right)\right]$$

$$r = \frac{E_c}{E_c - E_{sec}} \tag{13}$$

$$E_{sec} = \frac{f'_{cc}}{\epsilon_{cc}} \tag{14}$$

According to MANDER ET AL. [5] Young's modulus can be approximated by the empiric relationship: $E_c \approx 5000\sqrt{f'_{c0}} \quad [N/mm^2]$. A further simplification is given in PAPADOPOULOS AND XENIDIS [6] as $f'_{cu} \approx 0.8 * f'_{cc}$ to define the criteria for the ultimate strength of the confined core concrete. Originally in MANDER ET AL. [5] the point of ultimate strength (f'_{cu}, ϵ_{cu}) is examined by energy considerations and corresponds to the rupture of lateral reinforcement.

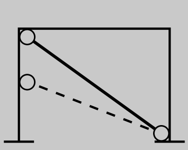
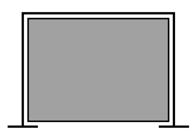
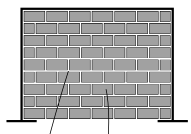
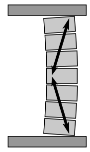
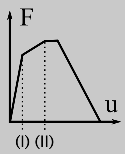
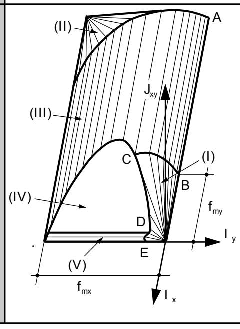
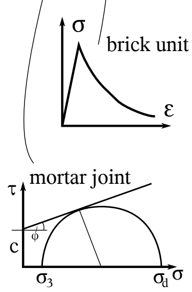
2.3 Three–Dimensional Discrete Modeling of RC Frame Members

The modeling is be done by distinguishing between spatial domains of composed materials. The common mechanical and damaging behavior of a structural element is defined by the single material behavior and interactions along its boundaries. A cascading approach on several levels is possibly and named multi–scale approach. To use a model on a higher level, a larger spatial scale, it is possible to complicate the constitutive relations to grasp more effects (e.g. concrete cracking) as illustrated in Table 1. A reasonable approach for RC frame member is to model the concrete apart form the reinforcement bars and to ensure that they interact with each other in the intended way using contact elements. An advantage of such strategy is that simpler constitutive relationships for each component can used to describe a complex structural element behavior. These constitutive relationships of the concrete and reinforcement bars can be obtained from numerical investigations on a smaller spatial scale. However, a general problem of discrete modeling arises. The gain over using relative simple material laws is payed by the need of knowledge of the location and the connection of different domains. On such small spatial scales, it is common and useful to describe the distribution of the different domains stochastically, but to perform a numerical simulation a defined distribution is necessary. So, to elevate the stochastic character up to a larger spatial scale, a sufficient number of calculations has to be performed resulting in huge numerical effort.

2.4 Three–Dimensional Continuum Modeling of RC Frame Members

Basis of the continuum concept is the assumption that the material properties can be averaged over a representative volume. Therefore it is only feasible to describe macroscopic phenomena of such a volume. The modeling of a structural element is executed by dividing it into a certain number of these representative volumes. Finally, the mechanical response of a structural element subjected to a load can computed by solving the differential equations describing this problem. Possibilities to grasp damaging effects are of either the use of plasticity models (with or without hardening properties) or alternatively damage material models with softening properties. In application an extension of this method considering damaging effects can lead to some difficulties such as numerical instability, mesh dependence solutions, increasing computational effort. Apart from that sufficient material properties are not as simple to determine to represent the reinforced concrete member in the intended way. In ANTHOINE ET AL. [7] a method is proposed take advantage of periodicity of the lateral reinforcement in RC frames to determine the homogenized properties of a member. At this the periodicity of the member is characterized by a basis cell corresponding to the distance between the lateral reinforcement. Since the member is uniformly loaded, two adjacent cells are subjected to the same loading conditions and thus deform in the same way. Considering the compatibility of the stress and strain conditions on their interface it is possible to derive the global behavior of the member and also its associated material properties.

Table 2: Models for brickwork infills in RC–Frames structures to capture damaging effects

	1D substitution	2D continuum	2D or 3D discret	out of plane
on element level				
on section level				–

The shaded models are used in the example.

2.5 One–Dimensional Substitute Brace Element

The infills are substituted by brace elements to consider their stiffening effect on the frame system approximately. The damage behavior of the infill shall be grasped by assigning non–linear softening material properties to the brace. Modeling structural elements by substituting them by systems differ in their load–bearing behavior is very useful in practical application when the limitations of the model are accepted. It is essential to know that the intention of substitution is to reflect the phenomenological behavior of the original system, not necessarily considering its physical mechanism. The relationship mostly is derived from observations obtained from laboratory test or field data analysis. An equivalent diagonal compression brace model to represent a masonry infill subjected to horizontal loads is proposed in FAJFAR ET AL. [8]. The brace behavior is approximated by a tri–linear force–deformation relationship, where each characteristic point is assigned to a damage state. F_c, u_c corresponds to the beginning of shear failure (bed joint sliding) and F_u, u_u corresponds to the initial stage of compression failure (corner crushing), see Fig. 2. Apart from the damaging process of the infill caused by deformations of the surrounding frame, the direct interaction between infill and surrounding frame is only considered as an additional contribution to the stiffness. However, the damaging process of the frame may be affected by the change of stiffness, which changes the global behavior of the building and in this way the effect of the impacting load on the frame. The initial stiffness K_i of the compression is the following:

$$K_i \quad = \quad \frac{E_p \, w_{ef} \, t_p}{r} \tag{15}$$

where E_p is Young's modulus of the infill panel in vertical direction. The empirical estimated effective area of the compression zone in the panel is described by w_{ef} and t_p, the panels thickness, see Fig. 2.

$$w_{ef} \quad = \quad 0.175 \, (\lambda_h \, h)^{-0.4} * r \tag{16}$$

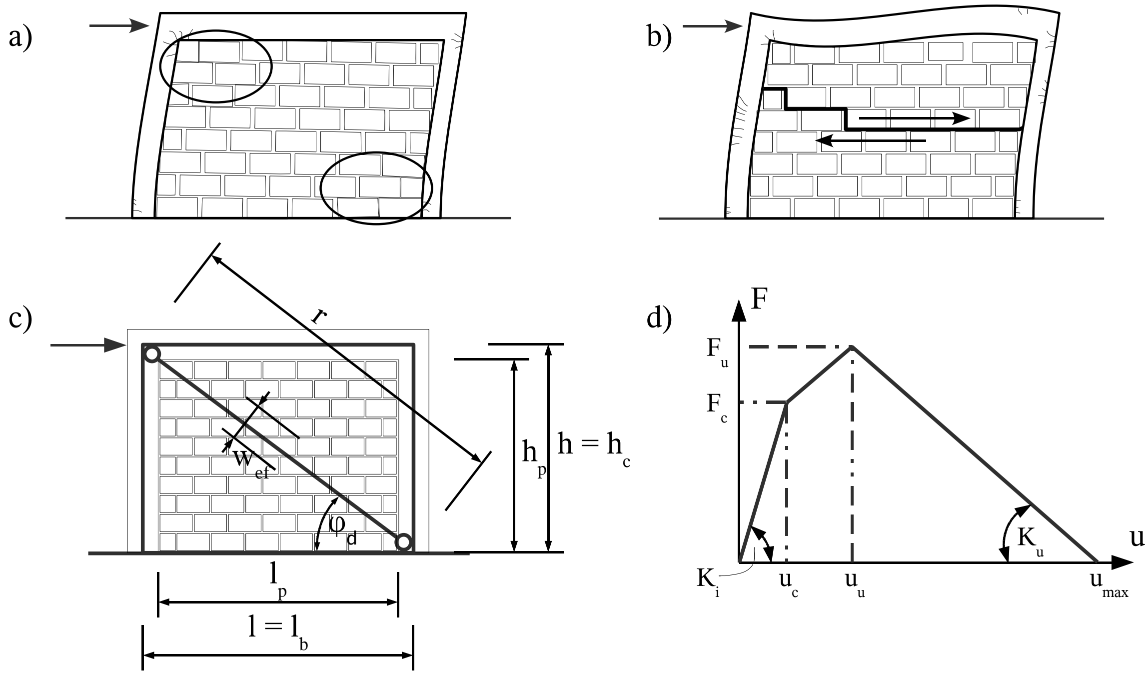


Figure 2: Typical damage mechanism for masonry infills: corner crushing (a) and bed joint sliding (b). Geometry of the substituted brace element (c) and the schematically force–deformation relationship (d) according to FAJFAR ET AL. [8]

$$\lambda_h = \sqrt[4]{\frac{E_p\, t_p\, sin(2\varphi_d)}{4\, E_f\, I_f\, h_p}} \tag{17}$$

$$\varphi_d = arctan(h/l) \tag{18}$$

The fracture force F_u of the brace can be determined by Eq.19. The parameter f_{tp} stands for the crack strength of the panel, its value can be obtained from the so called diagonal compression test (cf. FAJFAR ET AL. [8]).

$$F_u = 0.818 * \frac{l_p\, t_p\, f_{tp}}{C_1} * \left(1 + \sqrt{C_1^2 + 1}\right) * \frac{1}{cos\, \varphi_d} \tag{19}$$

$$C_1 = 1.925 * \frac{l_p}{h_p} \tag{20}$$

$$u_c = F_c/K_i \tag{21}$$

According to FAJFAR ET AL. [8] the corresponding brace shortening value can be calculated using $u_u = u_{u;\, horizontal}\, (cos\, \varphi_d)^{-1}$ assuming $0.005\, h < u_{u;\, horizontal} < 0.006\, h$. The cracking force, F_c, of the brace shall be in the range of $0.45\ F_u < 0.55\ F_u$. The corresponding deformation is shown by Eq. 21. The softening stiffness K_u can be assumed in the range $-0.10\, K_i < K_u < -0.05\, K_i$.

2.6 Two–Dimensional Continuum Modeling of Infills

Two dimensional shell elements are suitable to model in–plane loaded infills. This requires that the formulation of the failure criteria of the material captures the behavior of a brickwork infill as anisotropy, however such modeling does not consider the out–of–plane behavior. Because of the contact length along the entire infill boundary, a direct damaging interaction between both, the infills and the frame, is provided by the model. Therefore, additional shear forces along the length of columns and beams are considered, and these forces may lead to additional damaging stress conditions on adjacent frame members. Infill is a heterogeneous component. Thus it has multiple critical modes of failure, and such modes are to be included in the material model. A plastic model proposed in GANZ [9] has become popular in the last years, which consider such multiple damage modes. This model uses a combination of yielding surfaces, where each surface captures one failure mode. Table 2 shows these failure surfaces in detail. The failure modes are: tensile failure of the bricks (I), compression failure of the bricks (II), shear failure of the bricks (III), sliding along the mortar beds (IV) and tensile failure in the mortar beds (V). Only four independent input parameters are needed to define the material law, the strength in the main directions, the internal friction angle and the cohesion in the mortar beds.

2.7 Discrete Modeling of Brickwork Infills

The behavior of a heterogeneous brickwork is predominantly determined by the interaction of both components of the brickwork, brick and joint. Furthermore, the failure mechanism strongly depends upon amount and type of loading. Infills are considered to resist compression and shear forces, but not tensile forces as their adhesion strength in the joints is negligible. Moreover, there is a significant difference in stiffness between bricks and mortar, which leads to a failure of the infill caused by three–axial stress conditions. The behavior of the infills in resisting the applied forces depends on the geometry of the infills, the alignment of the bricks, the thickness of the mortar and the material description of the brick and mortar. Therefore, a discrete model is important to consider the combined effect of the geometry and the material of the brickwork infills. In such a model, the mortar is described by Mohr–Coulomb shear failure criteria and the bricks are described by a general damage model (cf. SCHLEGEL [10]).

2.8 Model for Out–of–Plane Behavior of Infill Walls

The predominant interaction between RC frame and infill is characterized by effects in plane direction. In three dimensional structures, infills are subjected to loads perpendicular to their plane, and the in-plane and out-of-plane forces interact with each other. Therefore, one or two dimensional models can not grasp out of plane failure modes. In order to consider out of plane behavior and assess its effect on the in-plane response in one or two dimensional models, a separate model is needed for the out of plane behavior. Such separate models are not required for three dimensional representations of the infills. The separate model presented by PRIESTLEY [11], describes the out of plane response of unreinforced infill subjected to uniformly distributed transverse, out of plane, forces. In this model the infill behavior is explained by an arch mechanism. A single joint failure is assumed in the middle of the short direction of the infill, which allows the rotation of the two resulted separated segments. These infill segments are idealized as truss elements connected by the plastic hinge (joint failure point) as illustrated in Tab. 2. The failure modes considered in this model are: loss of stability due to excessive rotation of the truss elements and/or the loss of strength of the infill's brick.

3 EXAMPLE

It is the purpose of the test setup to obtain a horizontal force–deformation relationship of the structure as a whole. For this the magnitude of a horizontal loading in accordance with a certain predefined pattern is incrementally increased. Here, uniformly distributed load is applied to all floor levels as illustrated in Fig. 3.

The tested structure is a partial infilled three story RC frame. Its infills have no openings and made from vertically perforated bricks with a thickness of 11.2 cm. Construction details as lateral reinforcement and the thickness of the concrete cover are not explicitly mentioned. But the given information indicates that the RC frame was built in consistence with the recommendation of the EUROCODE 2 [12] and EUROCODE 8 [13], therefore, the values for the lateral reinforcement of Ø 8; s=10 cm can be used. A concrete cover of 2 cm for the lateral reinforcement is obtained from the following graphs given by DOLŠEK AND FAJFAR [1]. The beam–column joints are assumed elastic during the test, their stiffness is equal to the corresponding concrete cross section. The test evaluation in DOLŠEK AND FAJFAR [1] has shown that the joints are kept nearly undamaged during the test, which confirms with the assumption of keeping them elastic. For further simplification, the assigned length of these joints is chosen as 20 cm measured by the static system, which correlates with the real geometry of the frame. All remaining structural elements of the reinforced–concrete frame are modeled as one–dimensional continuum elements on element level, whereas, on section level they are described by a discrete approach as illustrated in Tab. 1 and explained in section 2.2. It is assumed that the frame is fully restrained at the base.

The material parameters shown in Table 3 are taken from DOLŠEK AND FAJFAR [1] and they are the mean of the given values in the mentioned work. To calculate the material properties

Table 3: Considered mean material parameters taken from DOLŠEK AND FAJFAR [1].

Parameter	Symbol	Value
Concrete compression strength	$f_c; f_{c0}$	$3.60\ \mathrm{kN/cm^2}$
Concrete tension strength	f_t	$0.3\ \mathrm{kN/cm^2}$
Reinforcement yield strength	f_y	$55.5\ \mathrm{kN/cm^2}$
Masonry panel tension strength	f_{pt}	$0.048\ \mathrm{kN/cm^2}$
Vertical masonry Young's modulus	E_p	$690\ \mathrm{kN/cm^2}$

for the effective confined concrete Eq. 1 to 14 are considered. Regarding the ultimate compression of the core concrete f'_{cu} the relation published in PAPADOPOULOS AND XENIDIS [6] is applied. Because of the negligible differences of the columns material properties due to different reinforcement configurations, all columns are assumed to have the same material properties. As a result, the effective confined concrete of the column sections have the following attributes: $E_c = 3000\,kN/cm^2$, $\varepsilon_{cc} = 0.0048$, $f'_{cc} = 4.6\,kN/cm^2$, $\varepsilon_{cu} = 0.0133$ and $f'_{cu} = 3.68\,kN/cm^2$. The stress–strain relationship of the non–confined area of the cross section, the cover concrete, is the same as mentioned in the EUROCODE 2 [12]. This material law is also applied for the core section of the beams. The values are: $E_c = 3000\,kN/cm^2$, $\varepsilon_{c0} = 0.0022$, $f_{c0} = 3.6\,kN/cm^2$ and $\varepsilon_{(f=0)} = 0.0044$. The tensile strength of the concrete is taken into account by $f_t = 0.25\,kN/cm^2$ where $\varepsilon_t = 0.00025$ for all section areas. As aforementioned all beam–column joints behave elastically and have an assigned Young's mod-

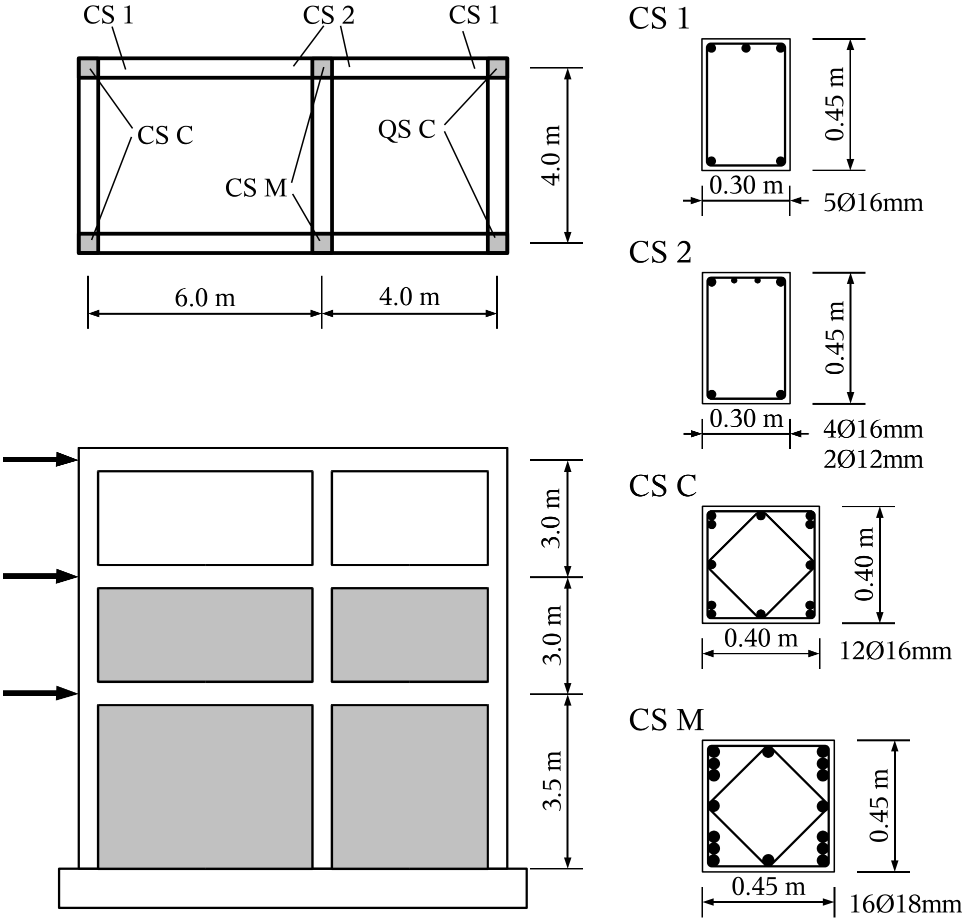


Figure 3: Example structure; taken from DOLŠEK AND FAJFAR [1]

ulus of $E = 3000\,kN/cm^2$. Based on the given information about the yield strength of the reinforcement, the following parameters can be derived: $f_y = 55.5\,kN/cm^2$, $\varepsilon_y = 0.00264$, $f_u = 65.0\,kN/cm^2$ and $\varepsilon_u = 0.1$, which are valid under tension as well as compression. The behavior of the compression brace elements substituting the infills is determined by using Eq. 15 to 21. The significant points that define their force–deformation curve have the following values. Brace 3x4m: $F_{c;\,3x4m} = 112\,kN$, $u_{c;\,3x4m} = 0.13\,cm$, $F_{u;\,3x4m} = 224\,kN$, $u_{u;\,3x4m} = 0.94\,cm$ and $u_{max;\,3x4m} = 4.3\,cm$; Brace 3x6m: $F_{c;\,3x6m} = 138\,kN$, $u_{c;\,3x6m} = 0.15\,cm$, $F_{u;\,3x6m} = 275\,kN$, $u_{u;\,3x6m} = 0.84\,cm$ and $u_{max;\,3x6m} = 4.9\,cm$; Brace 3.5x4m: $F_{c;\,3.5x4m} = 127\,kN$, $u_{c;\,3.5x4m} = 0.15\,cm$, $F_{u;\,3.5x4m} = 254\,kN$, $u_{u;\,3.5x4m} = 1.14\,cm$ and $u_{max;\,3.5x4m} = 5.1\,cm$; Brace 3.5x6m: $F_{c;\,3.5x6m} = 149\,kN$, $u_{c;\,3.5x6m} = 0.17\,cm$, $F_{u;\,3.5x6m} = 298\,kN$, $u_{u;\,3.5x6m} = 1.01\,cm$ and $u_{max;\,3.5x6m} = 5.7\,cm$.

Fig. 4 shows the allocation and dimensions of the modeled structural elements. Because of symmetry only one bay was considered, the stiffness of the lateral beams was neglected, since these beams rotate freely at both ends and show no resistance to the applied forces. An effective width of the slab is considered with the beam cross section (CS–1/2) in resisting the applied forces. The slab contribution is assumed to be 70 cm with a thickness of 10 cm. For simplification, a constant configuration of the reinforcement in the beams is assumed. The vertical reinforcement is taken constant over the height of the frame's columns. The column–beam joints are modeled as elastic beam elements with a length of 20 cm starting at the joint

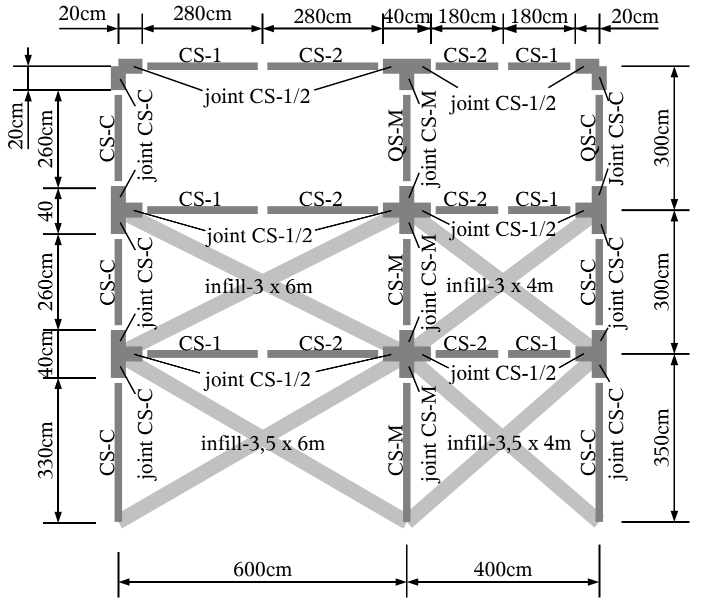


Figure 4: Allocation and dimensions of the modeled structural elements

intersection. The cross section dimensions are the same as the adjacent elements (beam or column). Compression brace representing the infill is a diagonal component connecting the intersection points of the frames's columns and beams. To apply the different material laws to the belonging areas of the cross section, the discretization of the columns and beams is done by so called fiber cross sections. A limited number of areas with their location over the cross section is defined. These fibers are 'rigid connected' according to Bernoulli–Navier hypothesis.

The dead load is calculated using a density of $25.0\,kN/m^3$ for the reinforced concrete members. The weight of the lateral beams and the slabs are distributed equally between the adjacent joints and beams. The additional load of the infills has a value of $3.43\,kN/m$, which is applied to the subjacent beam.

The simulation is performed by non–linear static analysis, where the vertical load is applied first, and afterwards the horizontal load (push–over load) is applied incrementally till a building drift of 0.05 is reached. During the calculation, the shape of the horizontal load is kept constant. As a result the force–deformation relationships (horizontal capacity) of the bare and the infilled frame systems are obtained. Fig. 5 and Fig. 6 shows the simulation results compared with the published ones in FAJFAR ET AL. [8]. In Fig. 7 the difference between the simulation and the test results. It can be said that the simulation of the RC-Frame with the applied models of the structural members is satisfactory. The differences between the simulation and the test results are less than 20 % for roof displacements less than 10 cm. which is apromising result. Without infills the maximum difference of 20 % is not exceeded till a roof displacement of 25 cm. Certain points (I to IX) on the curves can correlate directly with deformation states of the cross section fibers. The governing cross section fiber is located at the line between core- and cover concrete and the center of the reinforcement bars.

- Tensile strength in all columns is reached (I)
- Yielding of first main reinforcement bar (II)
- Spalling of concrete cover (III)
- Rupture of the first main reinforcement bar (IV)
- Tensile strength in all columns is reached (V)
- Maximum strength of the core concrete in one member is reached (VI)
- Rupture of the first main reinforcement bar (VII)
- Initial shear failure in one infill (VIII)
- Initial compression failure in one infill (IX)

In general, the calculated stiffness of the structure without infills match these of the tested ones, but the first loss in stiffness occurs later in the simulation, Fig. 5, (I). There are two possible explanations. Firstly, the assumed value of the tensile strength of the concrete is too high in the simulation. Secondly, the location of point (I) depends on the normal stresses acting on the section and this could lead to differences between the simulation and the test results. However, the load distribution of the test is known and well applied in the simulation. Thus the first possibility is more probable. This effect is also recognizable in the capacity curve of the infilled frame (cf. Fig. 6). Additionally, the elastic stiffness of the simulated and the tested frame results differ by $240\,kN/cm$, which indicates that the sum of the infills initial horizontal stiffness taken into consideration is higher by a value of $240\,kN/cm$. Meanwhile the stiffness of the RC frame is equal to the bare one. The displacement corresponding to the maximum base shear force of the infilled RC frame is $\approx\ 7\,cm$ and fits the test value relatively good. Therefore the non–linear deformation properties of the brace are relatively good determined.

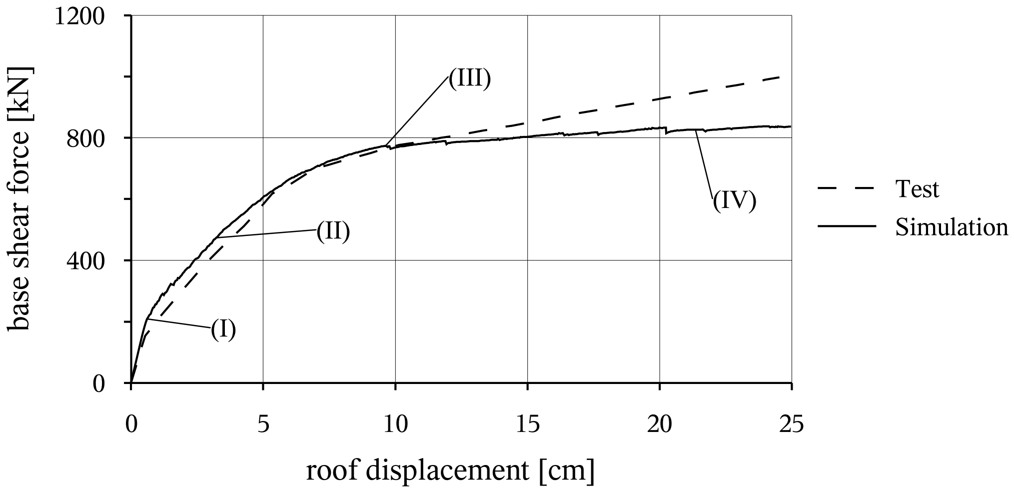


Figure 5: Comparison of the horizontal capacity of simulated bare frame with the results published in FAJFAR ET AL. [8]

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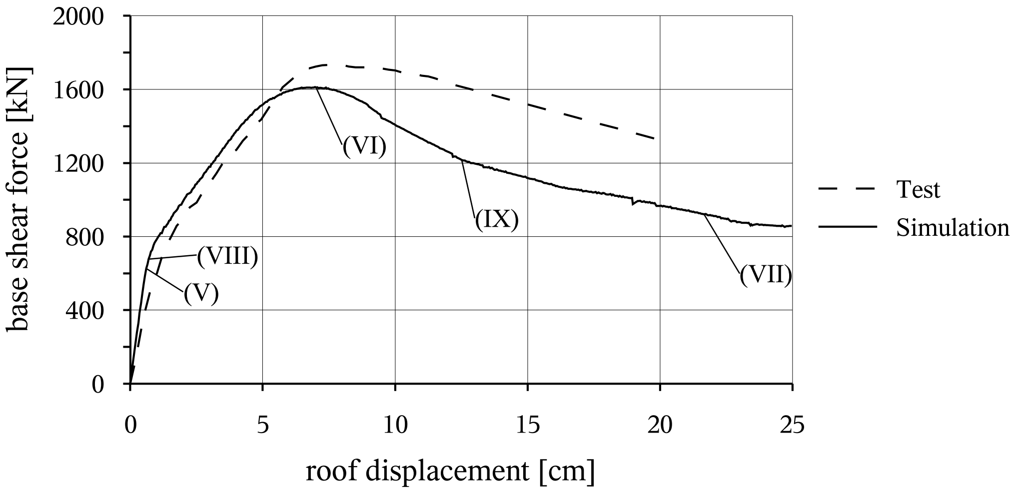

Figure 6: Comparison of the horizontal capacity of simulated infilled frame with the results published in FAJFAR ET AL. [8]

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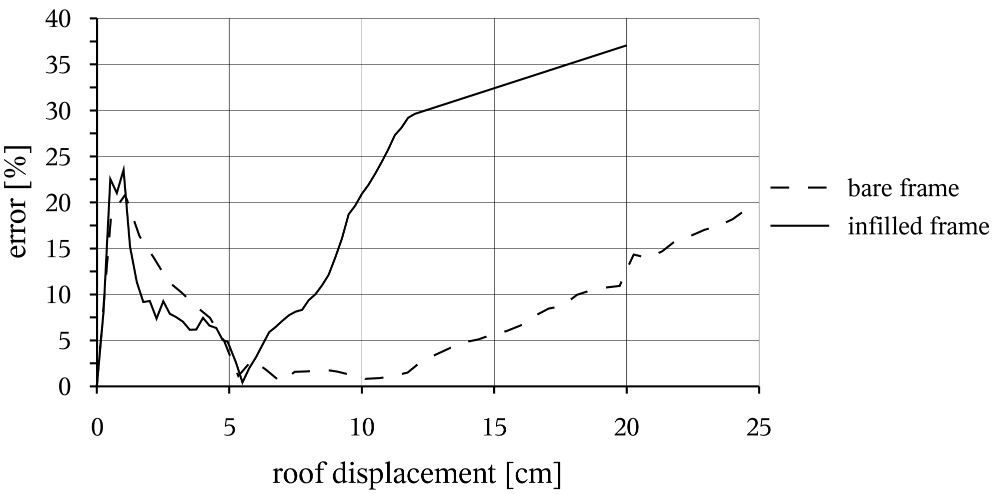

Figure 7: Comparison of the deviation of own results with the published ones.

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4 CONCLUSIONS

During the collation and analysis of the present models, it became clearer that there are many possibilities to model RC frame members. The reason may be that the models used in engineering applications aim to represent very specific aspects of a structure. However, it seems useful to apart such models into two categories. Firstly, the models developed directly from observations with the aim to represent the phenomenological behavior of a system, for instance the substituted brace model for the masonry infill. Secondly, models which try to explain the phenomenological behavior by analyzing the mechanism, for example the uniaxial stress–strain model for confined concrete. Both approaches are able to provide reasonable results as shown in the example explained previously in the paper. Nevertheless, such a conclusion can not be generalized, as its applicability is limited to the explained example where measured data were available. Further studies should focus on speeding up the simulation to be able to test different models for complete building structures. A promising possibility is trying to encapsulate and build the different models in a common framework and test their responses under unique conditions. The advantage of such framework is that the macroscopic behavior of the structural models can be determined and evaluated independently. Furthermore, the description of the macroscopic behavior can be used directly as a constitutive relationship in the simulation. Finally, an evaluation procedure for the models is needed, however, an evaluation or an assessment procedure is nothing but a model, consequently the results of an evaluation model depend on the chosen strategy. This aspect require an understanding of the purpose for evaluation of the different models.

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